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Earthquake failures of welded building connections

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Abstract

This paper presents the circumstances by which a flawed connection came to be used in tens of thousands of buildings of modern steel construction in seismic zones. Following the 1994 Northridge, California earthquake, it was found that these welded moment connections could not withstand expected earthquake distortions, and in fact fractured well below the intended design levels. The problem did not result from simple design or construction mistakes, but ultimately from an accumulation of evolutionary "refinements" in the design, materials and fabrication methods. This paper sheds light on gaps in the cooperative process between academia, professional engineers, and material and equipment providers that have led to tens of billions of dollars in potential repair/retrofit of modern buildings located in seismically active zones. Although millions of dollars have been spent in analyzing and testing this "simple" connection, little consensus has emerged regarding the extent to which each causal variable has contributed to the cracking. © 2001 Elsevier Science Ltd. All rights reserved.

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1. Introduction

Each year, major disasters involving engineered structures take a significant human and economic toll around the world. Among the principal responsibilities of the engineering community is to continually scrutinize the performance of these structures and to promote corresponding refinements in design methods, construction practices and material selection to better resist the next challenge. The process is iterative, combining failure analysis with corresponding corrective actions (Petroski, 1994). Examples include the collapse of dams, from which the resulting toll is many times worse than any conceivable flood the dam was meant to control. Bridge engineers have historically stretched out longer and longer spans, their progress occasionally slowed by the spectacular failures of premier structures. Structural failures during building fires were prevented by asbestos fiber fireproofing, now known as a carcinogen. Low-income shelters built of inherently brittle adobe and stone continue taking their annual toll in seismic zones around the globe. Many modern, technical designs have similar histories.

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While continual refinement undoubtedly leads to a better understanding of structural dynamics and material behavior, and consequently tougher buildings, mistakes and failures can still be expected. The January 1994 earthquake in Northridge, CA triggered the beginning of another such cycle, as building owners discovered that dozens of steel frame buildings, which they were told represented the state-of-the-art in aseismic design, suffered severe cracking. In fact, prior to the Northridge earthquake, welded steel moment resistant frames enjoyed the full trust of construction communities in the world's most advanced seismic areas of Asia and California. These structures were assumed to be strong enough to resist the stresses, and ductile enough to accommodate the distortions generated by severe earthquakes. Instead, the connection between the beams and columns fractured at load and deformation demands well below those for which they were intended. This paper shows how the initial concept, while shown to be reasonably successful in early laboratory testing and development, was eventually proven fatally flawed as the state-of-the-practice evolved into something different from the prototype.

2. Steel moment resisting frame history

The type of structure that is the subject of this paper is the steel moment resisting frame or SMRF. The frames are characterized by rigid connections between the beams and columns that force the entire frame to deform when subject to lateral load. In theory, under intense lateral loads resulting from seismic ground motion, energy should be dissipated in a stable manner as the frames distort. For severe seismic events, the beams would undergo substantial inelastic deformations, but the building should not collapse, thus, protecting the lives of occupants. The concept was popularized by the architectural desire to increase the space unobstructed by braces and shear walls in buildings, leading to steel frame construction with large open bays and correspondingly large structural members.

SMRFs have been used to resist lateral loads from wind and earthquakes since the turn of the century, when partial connection rigidity was accomplished through hot riveting, and later, bolting. Relatively recently, in an effort to ease the constructability of these structures (that is, lower the construction costs), a beam-to-column connection was developed based on full penetration welds between the beam and column flanges. Use of welded steel moment resisting frames in the construction of commercial buildings in seismically active regions has been a common practice since the early 1970s. SMRF technology is used around the world; it has been estimated that there are some 20,000 SMRF buildings on the west coast of the US and Canada alone.

Since the conception and original testing of welded SMRFs, several aspects of steel frame building design have changed. First, typical beam and column sizes in buildings have increased, and as such, fewer frames per building are needed to resist the lateral loading. Consequently, structural redundancy has been reduced. Secondly, flux-cored arc welding (wire) has essentially replaced shielded metal arc welding (stick) for all field welding of steel moment connections. The switch to wire changes the mechanism and rate by which the weld metal is deposited, at the same time delivering a substantially more brittle weld metal. The parent steel has also changed with time, as yield strengths have crept upward (particularly for steel delivered to meet minimum ASTM A36 requirements). All these changes were being introduced in an evolutionary effort to lower the cost of SMRF buildings. Because each change was done in an evolutionary and fragmented manner, little additional laboratory testing was triggered to assess the individual changes, let alone their combined effect.

While thousands of SMRF buildings were being constructed, limited testing of the connection and further refinement of the connection details continued. Although some problems were noted in the laboratory performance of the connections, most engineers and researchers continued to trust the connection would perform as intended. Most notable is the work done at the University of Texas, Austin in the

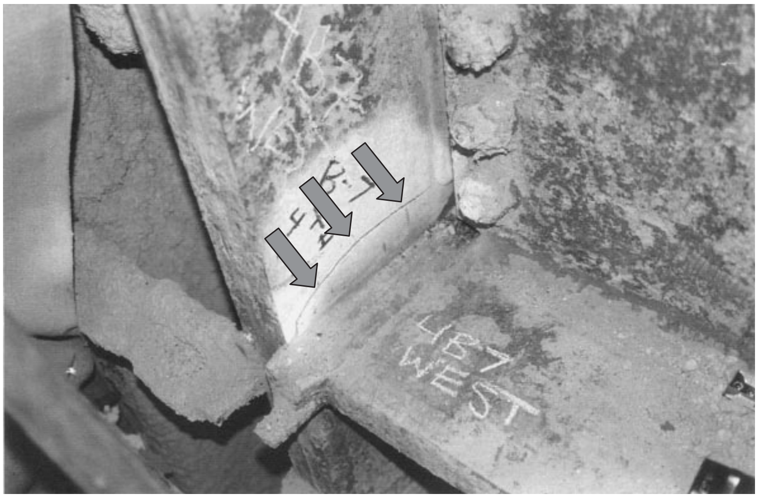

Fig. 1. Divot type connection crack in large section with no gross plastic deformation.

early 1990s, where premature cracking of SMRF connections during tests was widely published (Englehart, 1991, 1993). The engineering community met the implication that there would be widespread cracking of SMRF buildings in an earthquake with skepticism (Collin, 1992). Eventually, the real-world test by the Northridge earthquake demonstrated behavior that was markedly different from the design intent (Fig. 1) – connection fracture with little or no gross plastic deformation. Building owners were faced with a number of immediate and overwhelming dilemmas. Should the cracked connections be repaired to their pre-earthquake condition, only to crack again in the next earthquake? Is the risk of waiting to upgrade connections to an as-yet-undefined post-Northridge standard acceptable, or is immediate action required? What is an acceptable level of damage, in terms of dollar loss, in the next Northridge-like event?

Not surprisingly, the dramatic failures of seemingly "earthquake-proof" buildings has caused much excitement in the engineering, construction and commercial building owner community. The federal government has provided millions of dollars in research and testing of SMRF connections to determine what went wrong, what the best ways are to repair earthquake damaged SMRFs, and how best to design and build the next generation welded SMRF buildings that will perform reliably in the next earthquake (SAC, 1999a,b). After five years and tens of millions of dollars in research, surprisingly little consensus has been reached among engineers regarding the root causes of the cracking and most appropriate remedies.

One reason for the lack of consensus on causes and remedies is the wide array of crack topologies observed in buildings and laboratory test failures. However, only a few patterns of crack initiation and trajectory are evident. For instance, the cracks tend to initiate near the center of the beam-to-column flange juncture, at or near the root of the weld. After initiation, there are many different observed trajectories including propagation along the fusion line, into the base metal of the column, through the weld material, and occasionally confined to the beam base metal. For instance, cracks have been observed that have similar initiation and paths, and subsequently bifurcate to two or more trajectories, of which one becomes the dominant fracture surface (FEMA, 1997). In addition, the authors and others have observed instances of ductile tearing at the crack origin, which triggers a cleavage crack that then propagates in an unstable manner.

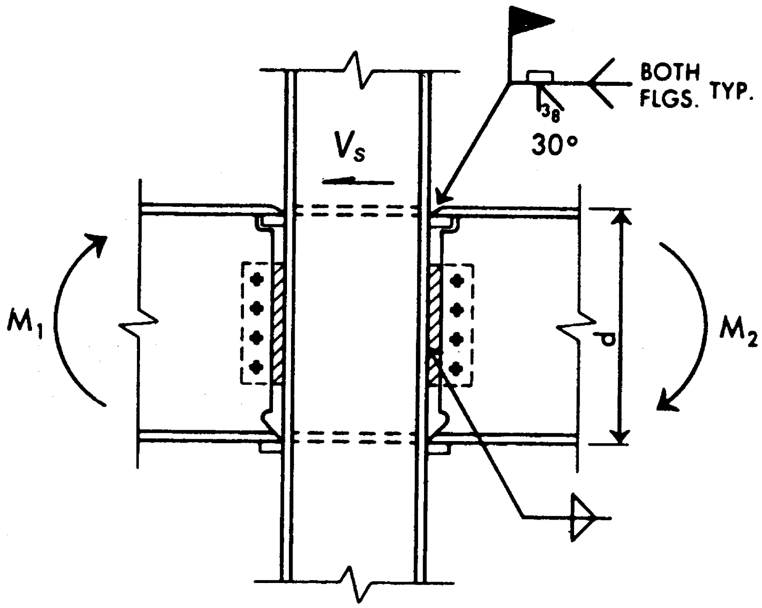

Fig. 2. Standard connection detail.

3. The connection

Virtually all west coast SMRF-buildings constructed in the last three decades have used the same standard connection detail (AISC, 1989) – welded flange and bolted web (Fig. 2). Until recently, all elements of the connection had been prescribed in detail by the building code, leaving little room for the individual engineer to deviate from the standard (ICBO, 1994). It was, in effect, a code-dictated rigid connection detail for steel frames. Acceptance of the moment connection resulted from many years of testing and research, primarily at University of California, Berkeley (Popov and Stephen, 1970; Roeder and Foutch, 1996).

Like virtually all structural connection details, the standard SMRF connection contains local stress risers, such as reentrant corners and bolt holes. However, designers have generally assumed that localized material yielding will redistribute the stress and mitigate the effects of such concentrations. Through redistribution, the typical SMRF connection was expected not only to carry the full moment capacity of the beam, but also to deform in a ductile manner well past its elastic limit. The welded joint between the column and beam flange was simply expected to transfer the associated stresses, without particular attention to the stress riser associated with the full-penetration, beveled groove weld joint detail. Properly detailed and fabricated, this weld configuration is considered "prequalified" under the building code based on a perceived history of good performance (AWS, 1994). Prequalified welded joints may be utilized without further, code-required testing to demonstrate adequate performance in a specific application. SMRF buildings constructed in the western US were typically welded using a continuous feed, self-shielded, flux-cored, wire electrode (AWS E70T-4). This process and electrode are popular for economic reasons – welds metal can be deposited very quickly, and in field conditions, particularly high wind, where other weld processes face difficulties. A key disadvantage of this process/material is a much lower notch toughness of the as-deposited metal compared to typical construction steels or weld metal deposited using common stick electrodes (E7018). For instance, investigators at Lehigh have measured E70T-4 weld material toughness (room temperature Charpy V-Notch) of 10 ft lbs, as opposed to 144 ft lbs for weld metal deposited by the

E7018 stick electrode. Many researchers believe that the low toughness weld deposit is a chief reason for the underperformance of the connection (Kaufmann et al., 1996; Uang et al., 2000).

The connection geometry, its fabrication, materials, and utilization in modern building design were born of a cooperative, overlapping development process that involved academics, structural engineers, metallurgist/welding engineers, material producers and fabricators/erectors. All contributed to the final form and application of the standard SMRF connection: academic researchers performed some tests on specimens that were generally smaller than those now used in construction, concluding that the connection could reliably sustain predicted loads and deformations without failure; electrode and welding equipment suppliers were continually refining flux core arc weld material and deposit processes in an effort to improve the competitive economy of welding yet the issue of appropriateness of the resultant weld deposit for buildings in seismic areas was not communicated with the designers; researchers' community and code officials extended the perceived history of good performance of the prequalified weld detail to jumbo sections subjected to extreme plastic deformations; designers relied on the code prescriptions and applied the weld to jumbo sections, consequently, reducing the total number of moment connections in a building; and the fabricators and erectors were drawn to the most quickly deposited, and thus most economical, weld process and material available in the market.

4. The suspect variables

The research to date has identified many technical factors that contribute to the SMRF connection damage (SAC, 1999a,b): standard use of weld materials that result in low notch toughness weld deposits; stress triaxiality due to connection geometry; high stress (strain) risers at geometric discontinuities; unreliable steel properties in the direction perpendicular to rolling; design practice that favors relatively few moment resisting frames; larger sections than had been previously tested; difficulties in non-destructive inspection of connections; excessively weak and flexible column panel zones; and highly variable and unpredictable material properties of beams and columns, particularly beam yield strength. In fact, the effort to understand this connection's behavior since its shortcomings were discovered in the Northridge earthquake is at least an order of magnitude greater than the total effort expended to develop and refine the design prior to 1994.

Notwithstanding the large number of field failure observations and millions of dollars of analytical and laboratory investigations, debate continues regarding the root causes of the failures. As mentioned, the debate is largely driven by the variability observed in the fracture mechanism from building to building and connection to connection. Also, fueling the debate is the inherent complexity of the analytic models required to capture the three dimensional, elasto-plastic fracture process. The analytic effort to date has failed to yield a model that accurately predicts the loads and deformations to fracture, much less the wide variety of crack trajectories. Without such models or an extensive and expensive testing matrix that methodically varies each variable, it is not possible to quantify the contribution of each potential causal factor. Nevertheless, promising research in those areas is progressing (Chi et al., 1997; Tawil et al., 2000).

Regardless of the crack origin and trajectories, all the observed fractures surfaces are dominated by brittle, cleavage fractures. This and the lack of conspicuous plastic deformation has led some investigators to assume that linear elastic fracture mechanics (LEFM) could be used to model, investigate and eventually predict the failures. Attempts with simplified LEFM models yielded limited or serendipitous success. In fact, many aspects of failures observed in laboratory and field specimens suggest that LEFM fails to capture important aspects of the failure process. Failures were found often to occur only following multiple load cycles at the same load (displacement) level; localized ductile tearing has been observed at some fracture initiation sites; elasto-plastic finite element analysis predicts insufficient confinement, leading to large-scale yielding around the crack tip, and laboratory connection strength does not correlate well with

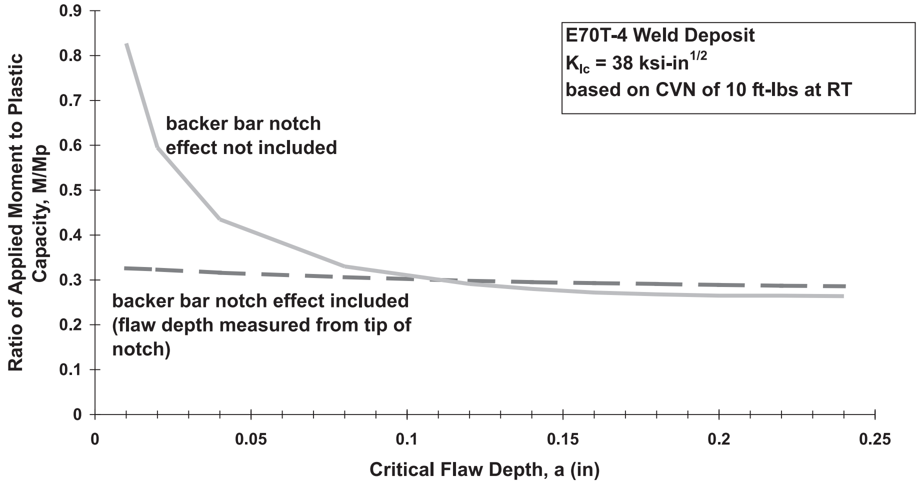

Fig. 3. Flaw size vs. fracture moment.

robust LEFM analysis using measured fracture toughness of the weld and parent steel materials (Chi et al., 2000). The authors applied LEFM to the 3D stress field determined by finite element analysis and predicted the critical fracture load for various assumed flaw sizes (Fig. 3). For commonly observed flaw sizes (Paret, 2000), the LEFM predicted failure load (30–40% of the beam plastic moment) is well below typically observed laboratory values (80–100% of the beam plastic moment). These results are similar to those given by Chi et al. (2000), who present potential reasons why LEFM underpredicts actual strengths: real weld flaws are not "mathematically" sharp, assumed flaws extend for the entire length of the flange, and the additional toughness provided by stable crack growth is neglected.

LEFM underpredicts measured connection strengths because it over-simplifies a more complex fracture process. Recent technical publications (Kuwamura, 1996, 1998; Ostertag, 1997) have described a failure mechanism that it consistent with both laboratory observations and stress analyses. Ductile tearing begins at a location of high cyclic plastic strain, and grows in a stable manner until conditions of stress and constraint a short distance ahead of the tear are sufficient to initiate an unstable cleavage fracture. This mechanism has been observed in tests conducted in Japan and confirmed by examination of structure steel connection failures that occurred during the 1995 Hyogoken–Nanbu earthquake (Kuwamura and Yamamoto, 1997). Using optical and scanning electron microscopy, the authors have observed instances of very localized ductile tearing at the crack origin of several SMRF crack samples removed from Northridge area buildings (Fig. 4), and believe that further research of this mechanism and how it relates to SMRF cracking is warranted.

5. Summary

This paper summarizes the failure of steel moment connections designed to resist substantial earthquake-induced distortion without fracture. The acceptance of promising, early research results, and the subsequent extrapolation to large member sizes, more brittle materials, new joining techniques, and less redundant structural system configurations led to an unexpected, and devastatingly expensive, widespread failure of welded connections during the 1994 Northridge earthquake. The episode shows once more how the design/construction industry, by adopting innovative, "state-of-the-art" solutions without questioning

Fig. 4. Ductile tearing at root of predominantly cleavage fracture.

the popular design "wisdom," and the material manufacturer assurances, exposed itself to widespread, heavy consequential failures in the welded beam-to-column connections of steel frames. The burden of a continual vigilance to reduce the risk of such occurrences cannot, however, rest with any single party involved in the process, but has to be a source of open dialogue between the researchers, designers, contractors and material suppliers. Absent such vigilance, the cycle of catastrophic failure and subsequent engineering response will continue to be the costly way of progress.

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